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SUSPENDED SOLIDS REMOVAL

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Environmental Protection Agency
Cincinnati, Ohio

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INTRODUCTION

Suspended solids removal is a necessary consideration in all waste treatment operations. Solids which must be removed may be those present in the raw sewage, those added by some form of recycle within the plant, or those created by biological or chemical actions during treatment. Suspended solids, unlike many other pollutants, vary widely in their chemical and physical makeup. They, therefore, can and do act differently under different conditions imposed by various treatment processes and in different waste streams. Tertiary removal of suspended solids from a waste stream may also accomplish an increase in organic removal necessary to meet effluent standards due to the organic nature of some of the solids. Due to time limitations the following discussion will be limited to only three available tertiary processes which may be utilized for upgrading the efficiency of existing treatment facilities. These processes are microstraining, high-rate sedimentation, and granular in-depth filtration.

MICROSTRAINING

Microstraining has been a viable solids-removal process for twenty years. This discussion concentrates on the Crane-Glenfield Microstraining process, although other proprietary units are commercially available*. Descriptively, a microstrainer consists of a rotating drum with a screen or fabric constituting the periphery. Feedwater enters the drum internally and passes radially through the screen with the concomitant deposition of solids on the inner surface of the screen. At the top of the drum pressure jets of water are directed onto the screen to remove deposited solids. This backwash stream of dislodged solids and washwater is captured in a receiving hopper inside the drum and flows out through the hollow axle of the unit. The unit is illustrated in Figure 1. The drum sizes and capacities of commercially available units are shown in Table 1. Since the complete unit includes influent and effluent structures and housing, the overall space requirements are shown in Figure 2. In order to reduce slime growths on the screen an ultraviolet lamp is continually operated in close proximity to the screen. The driving force for the system is the head differential between the inside and outside of the screen. As solids are removed on the screen a mat is formed which improves the solids removal efficiency and also results in increased headloss through the screen. The maximum headloss is usually limited to 6 inches in order to prevent screen damage. In order to prevent the limiting headloss from being exceeded drum speed and washwater pressure are increased. In newer units automatic controls handle these adjustments. Screen specifications are noted in Table 2.

Results from a number of installations around the world are available in the literature. Some of the available data are collected in Table 3. It should be noted that suspended solids removal is the parameter of primary

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interest, and BOD removals are only a function of the makeup of the solids removed. Mark I screens display efficiencies from 55 to 73 percent, while Mark O shows 57 to 89 percent efficiencies in this tabulation of average removals. Due to differences in feed character and operational techniques the data cannot be compared. Individual studies demonstrated the effects of a number of design, maintenance, and operational factors on the performance of the unit:

Design

1. Approximately one-half of the washwater applied to the screen actually penetrates and is removed as the waste stream with dislodged solids.
2. The waste stream is usually returned to the head of the main plant.
3. It is desirable to have gravity flow from the clarifier to the microstrainer to avoid shear of the more fragile solids.
4. Total headloss through the system is only 12 to 18 inches.
5. Prechlorination should be avoided in order to protect the screen.
6. Chloride concentrations exceeding 500 mg/l may cause corrosion problems.
7. Microstrainers do not successfully remove floc particles resulting from coagulation by chemicals such as aluminum sulfate.

Maintenance

1. Strainers for the pressure washing system tend to clog, mainly due to grease in the effluent.
2. Most units will require frequent (approximately once/week) cleaning with a hypochlorite solution which entails a few hours of removal from service in order to clean the fabric.
3. High iron or manganese concentrations in the feed may necessitate an occasional acid wash of the screen to destroy the resulting film buildup.

Operation

1. Minimum drum speeds consistent with headloss limitations will give the greatest removal of suspended solids.
2. Higher pressures for the jet washing system are more beneficial than greater quantities of water.
3. High solids loadings can cause severe reductions (up to 2/3 design capacity) in throughput as well as acceleration of slime buildup.

The cost of microstraining was estimated by Smith as depicted in Figure 3. Another estimate which was most carefully developed from the Chicago study for a 10 mgd plant is 2.86¢/1000 gal.

TUBE SETTLERS

Classical sedimentation theory has established that clarifier detention time has no direct effect on suspended solids removal efficiency. As illustrated in Figure 4, the combined impulse and gravitational vectors determine the ideal settler efficiency depending on depth. If a very shallow depth is provided, a minimum size tank is required. Utilization of this theory was impractical until very recently due to highly irregular flow patterns and sludge removal problems from shallow-tray settlers. The "tube settler" concept has generally eliminated both of these problems. The Neptune Micro-Floc Company* developed this concept in the USA and the data herein reported refers to their installations. At least one other company has a competitive device which is commercially available.

An over-simplified vector diagram is presented in Figure 5 to illustrate what occurs in the tube. After striking the bottom surface of the tube, the solids must either accumulate until eventually clogging the tube or be removed. Tests of the angle of inclination of the tubes indicate that a 60° angle with the horizontal allows for sludge drainage freely down the bottom of the tubes, while still producing comparable effluent quality. The obvious advantage of this type of device is the reduction in required sedimentation area and its associated capital costs. This type of savings can be realized both in the design and construction of new plants as well as in upgrading the performance or expansion of existing facilities.

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Tube settler modules, depicted in Figure 6, have dimensions of 10 ft long, 30 in. wide, and 21 in. deep. Construction of the sidewalls is of PVC and the transverse, tube-forming panels are of ABS plastic. These modules have a bulk density of only 2.5 lb/sq ft of surface and have the structural strength required to consider them as beams requiring support only at their ends. The modules can be described as a series of 2 in. square tubes 24 in. in length at an angle of 60° from the horizontal. The material of construction allows for trimming to fit individual structures. Installation of these modules for expansion of existing facilities usually will require baffling of the tank to force all the liquid through the modules along with increased weir and pipe capacity to handle the additional flow. The modules are usually positioned so that their tops are two feet below the surface while their bottoms must provide sufficient clearance for sludge removal facilities.

Some of the operating experience available in the literature is shown in Table 4. Increased capacities with similar or better solids removal are apparent when the hydraulic loading or overflow rates are compared. Some problems concerning growth attachment to the tubes have been noticed. In most cases a monthly drawdown or surface spraying is required to clean the tubes. Some structural and maintenance difficulties have resulted from the use of two-deep modules. An objection has been raised to the reduced secondary clarifier surface requirements by one researcher. He points out that thickening of accumulated sludge in the secondary clarifier may possibly be impaired with resulting increased pumping requirements. To counter this argument it should be noted that the majority of thickening occurs in the first few minutes and that the problems associated with gasification in the secondary clarifier should be minimized.

In summary, although little cost information is available the manufacturer indicates that tube settler costs can vary from about 12 to 20 dollars/sq ft, and installation costs can vary from 5 to 15 dollars/sq ft. The higher unit costs are associated with smaller installations, but can vary with basin shape and other factors, also.

IN-DEPTH FILTERS

It would be imprudent, at best, to provide design data for filtration. Each installation not only has a different feed to be filtered, but also has individual requirements and limitations. In effect, theory has not satisfactorily served the requirements of present day practice. The situation is not as bleak as it may appear, however, as successful filters are being installed throughout the country. Filtration is capable of producing an excellent effluent in terms of suspended solids, with

concomitant removals of organic matter associated with these solids. The only system which is considered here is direct filtration of secondary effluent. When phosphorus removal is not a problem, it has been difficult to economically justify coagulation ahead of the filters in light of the small improvement in effluent quality.

It can be safely stated that any proposed use of filtration for tertiary suspended solids removal should be studied in the laboratory or pilot plant prior to design. In preparation for such testing the boundary conditions or limits of design freedom should be determined in order to prevent unnecessary efforts. These limits may be capital expenditures, operator availability, land area, flow variability or infiltration, or discharge conditions to name a few. Since the choice of designs and appurtenances is almost unlimited, this preparatory work can pay high dividends. Flow through the filter can be upflow or downflow. Driving forces can be pressure or gravity. Flow rates can be high or low as well as constant or declining. Media can be sand, coal, a combination, or a combination plus garnet. Backwashing can be accomplished with water at high and low rates, with a surface wash in addition to the normal, or with an air-water combination. This list can be expanded, accomplishing nothing except to further confusion.

The performance of the filter is affected by a number of variables which must be taken into account prior to pilot studies. These include the concentration and character of the applied solids, filter media size and design, flow rates, and backwashing techniques. The single most important factor in determining the success or failure of a filter is the solids in the feed. A high concentration of feed solids can plug a filter quickly causing shorter runs and high percentages of backwash water. The strength of these solids is related to the processes preceeding the filter. In general, activated sludge solids become trapped on the surface of the filter and resist shear and penetration into the bed while producing good quality effluents. Trickling filter solids are less amenable to filtration and produce a lower effluent quality. Some investigators have correlated activated sludge effluent filterability directly with aeration time and inversely with the organic load factor. As might be expected, larger particle sizes are generally more easily filtered.

The filter media also plays a large role in determining filter efficacy. In terms of size, smaller grains produce better effluents, while larger grains produce lower rates of headloss development. Some investigators have found increased depth to be beneficial at the expense of increased head requirements. Anthracite, owing mainly to its angular grains, generally reduces the rate of headloss development, sometimes at the expense of effluent quality deterioration. As illustrated in Figure 7,

any single type of media offers most of its flow resistance and removal ability at the top of the filter bed, with little or no productive activity in the remainder of its depth. Ideally, this situation would be reversed in order to provide continuously finer removal as passage through the bed progresses. Such a situation should provide long runs and low backwash requirements due to the distribution of floc storage throughout the bed. One way of approaching this situation is also illustrated and is called a dual-media filter. This concept can be extrapolated further by utilizing a third media such as garnet. The size distributions are maintained throughout backwashing due to the differences in specific gravity of the media. A more perfect simulation of the ideal filter is the upflow filter. Extensively used in Great Britain, it offers a number of advantages from the standpoint of run length and backwash savings.

Due to the previously noted strength of activated sludge solids, a number of investigators have noted that the rate of filtration has little effect on filter performance at hydraulic rates of 10 gpm/sq ft. It therefore appears somewhat questionable that present day designs are still being fostered with the old standard of 2 gpm/sq ft or similar values. Utilization or at least investigation of these higher flow rates can result in significant savings if they can be shown feasible in terms of effluent quality and operation.

As filter runs lengthen the percentage of filtered water needed for backwashing is reduced. In water treatment plants this is desirable; in waste treatment plants it can be a problem. If the primary purpose of the filters is to remove suspended solids, long filter runs can allow slime growths in the media which can impair this goal. These slimes are tenacious and can require special backwashing techniques and costly chemicals for cleaning. Some installations use twice daily backwashing to prevent these problems. In most cases surface wash systems are required. In many cases air-water backwashes have been used. The use of the air-water backwash has not been fully explored for dual or multi-media systems and should be viewed with a "you show me" attitude at present.

Many other considerations are involved in designing filters for tertiary solids and associated organic removals. These include such things as the choice of gravity or pressure systems. The latter provides for lower capital and installation costs as well as reduced space requirements, while the former provides for better access to the media for maintenance purposes and generally lower operating costs. Most of the major equipment manufacturers can provide the necessary units, but it is prudent for the consultant or client to know what the project needs are by laboratory or pilot studies before contacting them. A cost estimate for granular filtration is presented in Figure 8.

SUMMARY

In summary, three methods for upgrading suspended solids and associated organic removal have been discussed. These methods were primarily chosen for the purpose of improving existing plant performance. Other methods are available, but were not discussed due to the time limitations imposed.

REFERENCES

Bodien, D. G., and Stenburg, R. L.
"Microscreening Effectively Polishes Activated Sludge Plant Effluent."
Water & Wastes Engineering (September 1966).

Culp, G., Hansen, S., and Richardson, G.
"High Rate Sedimentation in Water Treatment Works."
JAWWA, 60, 681 (1968).

Diaper, E. W. J.
"Tertiary Treatment by Microstraining."
Water & Sewage Works, 202 (June 1969).

Gloyna, E. F., and Eckenfelder, W. W. Jr.
Water Quality Improvement by Physical and Chemical Processes
Publ. by University of Texas Press, Austin, Texas (1970).

Hansen, S. P., Culp, G. L., and Stukensberg, J. R.
"Practical Application of Idealized Sedimentation Theory
in Wastewater Treatment."
JWPCF, 41, 1421 (August 1969).

Misaka, Y., and Polkowski, L. B.
"Filtration of Activated Sludge Secondary Effluents Through
Sand and Anthracite-Sand Beds."
The University of Wisconsin Water Resources Center (June 1969).

Mueller, H. M., Jr.
"High Rate Sedimentation and Filtration." Part 1 and Part 2.
Water and Wastes Engineering, June 1970 and July 1970.

Truesdale, G. A., and Birkbeck, A. E.
"Tertiary Treatment Processes for Sewage Works Effluents."
Water Poll. Control Jour. (British) 66, 371 (1967).

TABLE 1
UNIT SIZES AND CAPACITIES

DRUM SIZES IN FEET		APPROX. CAPACITY
<u>DIAMETER</u>	<u>WIDTH</u>	<u>mg/l</u>
5	1	0.05 - 0.5
5	3	0.3 - 1.5
7-1/2	5	0.8 - 4
10	10	3 - 10

TABLE 2
CHARACTERISTICS OF THREE FILTER FABRICS

	<u>MARK O</u>	<u>MARK I</u>	<u>MARK II</u>
Apertures:	1/1000 23 microns	1.5/1000 35 microns	2.5/1000 60 microns
No. per sq. in.:	160,000	80,000	60,000
No. per lineal in.:	400	280	240

TABLE 3

TERTIARY TREATMENT BY MICROTRAINING

<u>LOCATION</u>	<u>FABRIC Mark</u>	<u>SIZE mgd</u>	<u>S. S. rem. %</u>	<u>BOD rem. %</u>	<u>BACKWASH %</u>	<u>FEED type</u>
Luton, England	I	3.6	55	30	3.0	COMB.
Bracknell, England	I	7.2	66	32	-	T. F.
Harpendon, England	I	0.3	< 80	-	-	T. F.
Brampton, Ontario	0	0.1	57	54	-	A. S.
Chicago, Illinois	0	3.0	71	74	< 3.0	A. S.
Lebanon, Ohio	0 I	pilot	89 73	81 61	5.3 5.0	A. S. A. S.

TABLE 4

TUBE SETTLER INSTALLATIONS

PLANT LOCATION	TYPE	SIZE mgd	TUBE LOC.	TUBE DATA		EXISTING FACILITY	
				O.R. gpm/sf	EFF. SS mg/l	O.R. gpm/sf	EFF. SS mg/l
Philomath, Oregon	TF ^a	0.15	Aerator ^a	3.3	43	-	-
Philomath, Oregon	TF	0.15	S. Clar.	3.3-4.6	60-70	0.6	60-70
Philomath, Oregon	TF	0.15	P. Clar.	2.1-3.3	34-41 ^b	0.84	40-45 ^b
Hopewell Twp., Pa.	AS	0.13	S. Clar.	2 - 3	27	0.34	60-70
Miami, Fla.	AS	1.0	S. Clar.	1.7	33	1.3	500
Ontario, Canada	AS	-	Aerator	1.0	28	-	-

a. Secondary clarifier converted to aerator + tubes

b. Percent removal rather than concentration

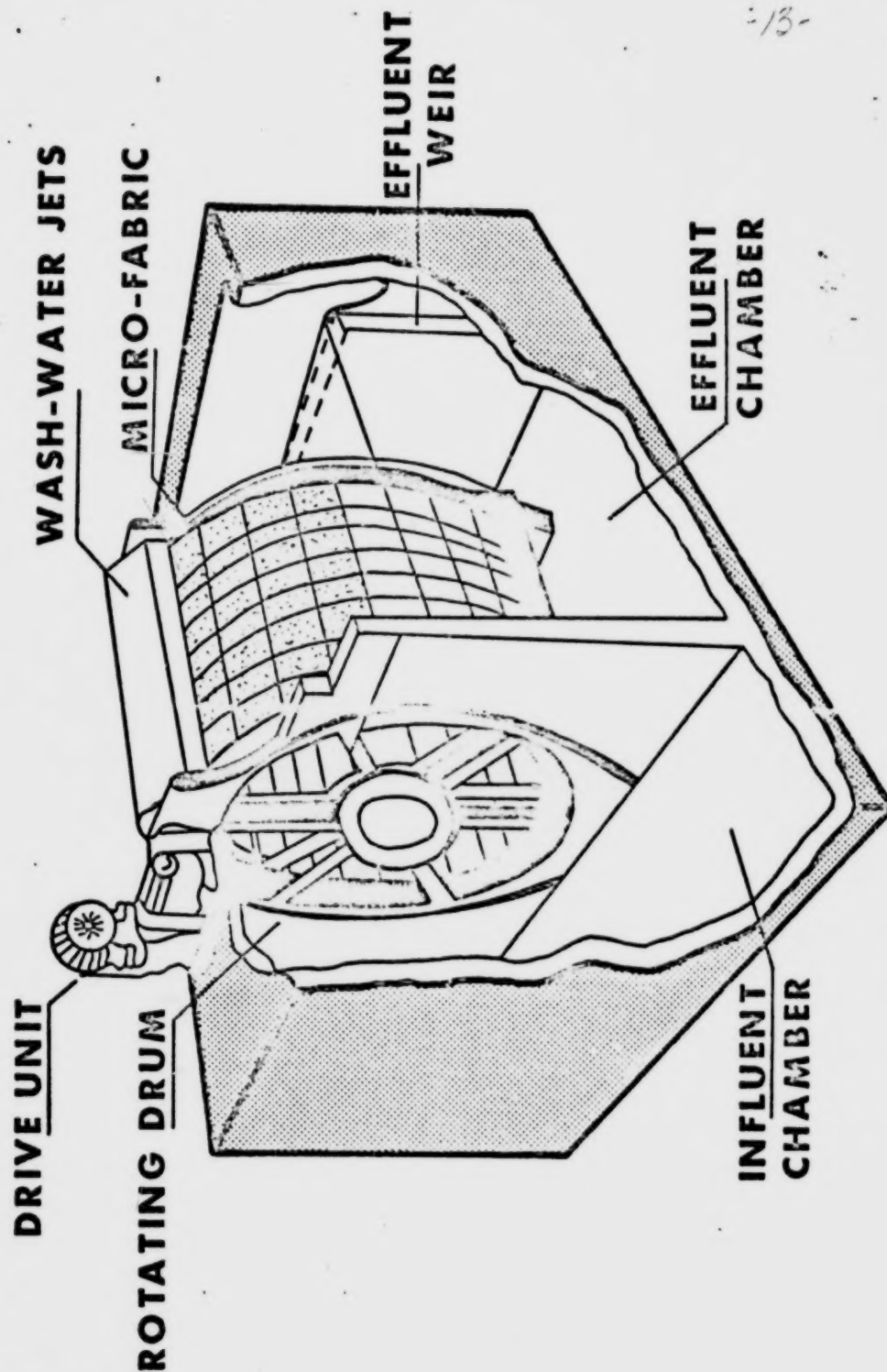
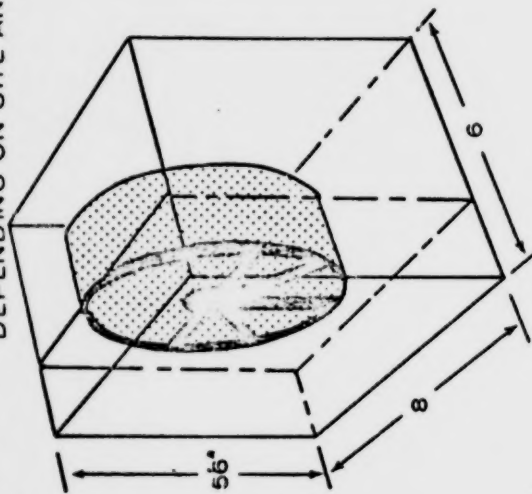
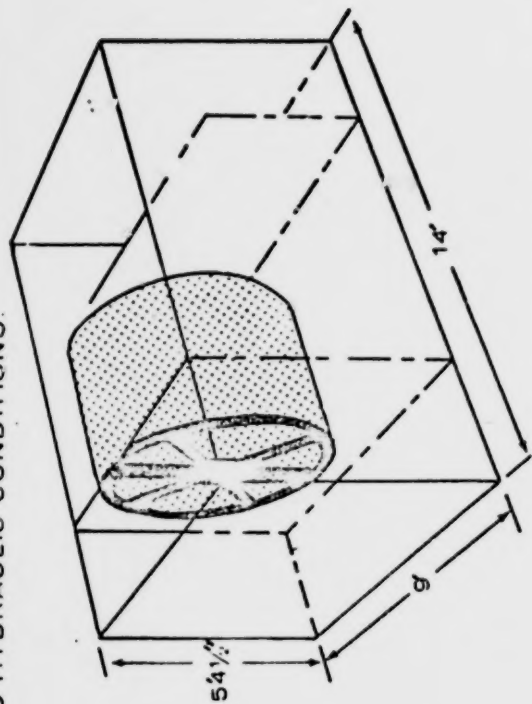


FIG. 1 **TYPICAL MICROSTRAINER UNIT**

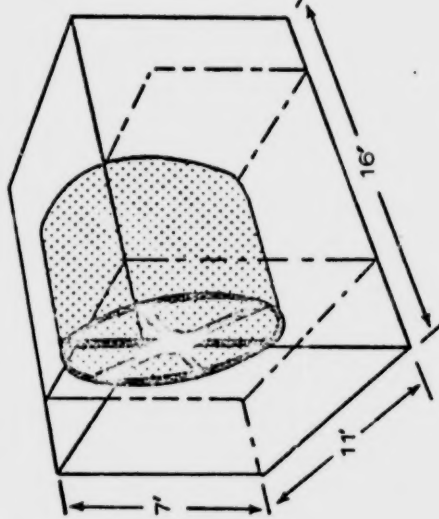
Fig. 2 ISOMETRIC SKETCHES SHOWING MINIMUM INTERNAL DIMENSIONS OF TANKWORK
THE DIMENSIONS ARE APPROXIMATE AND SUBJECT TO MINOR ALTERATION
DEPENDING ON SITE AND HYDRAULIC CONDITIONS.



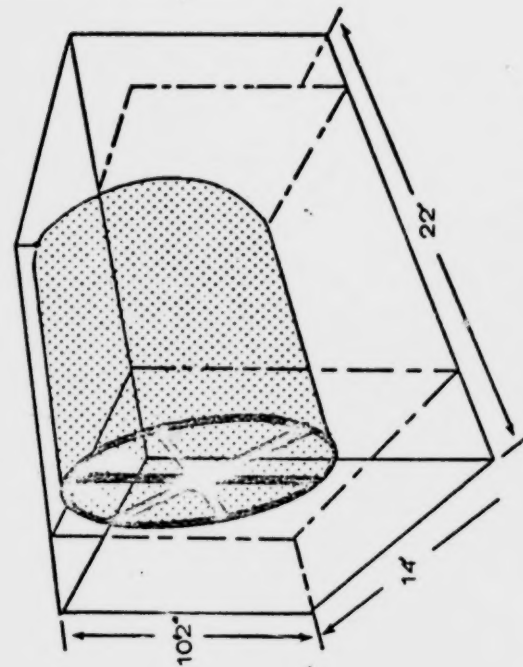
5' DIA. x 1' WIDE MACHINE



5' DIA. x 3' WIDE MACHINE

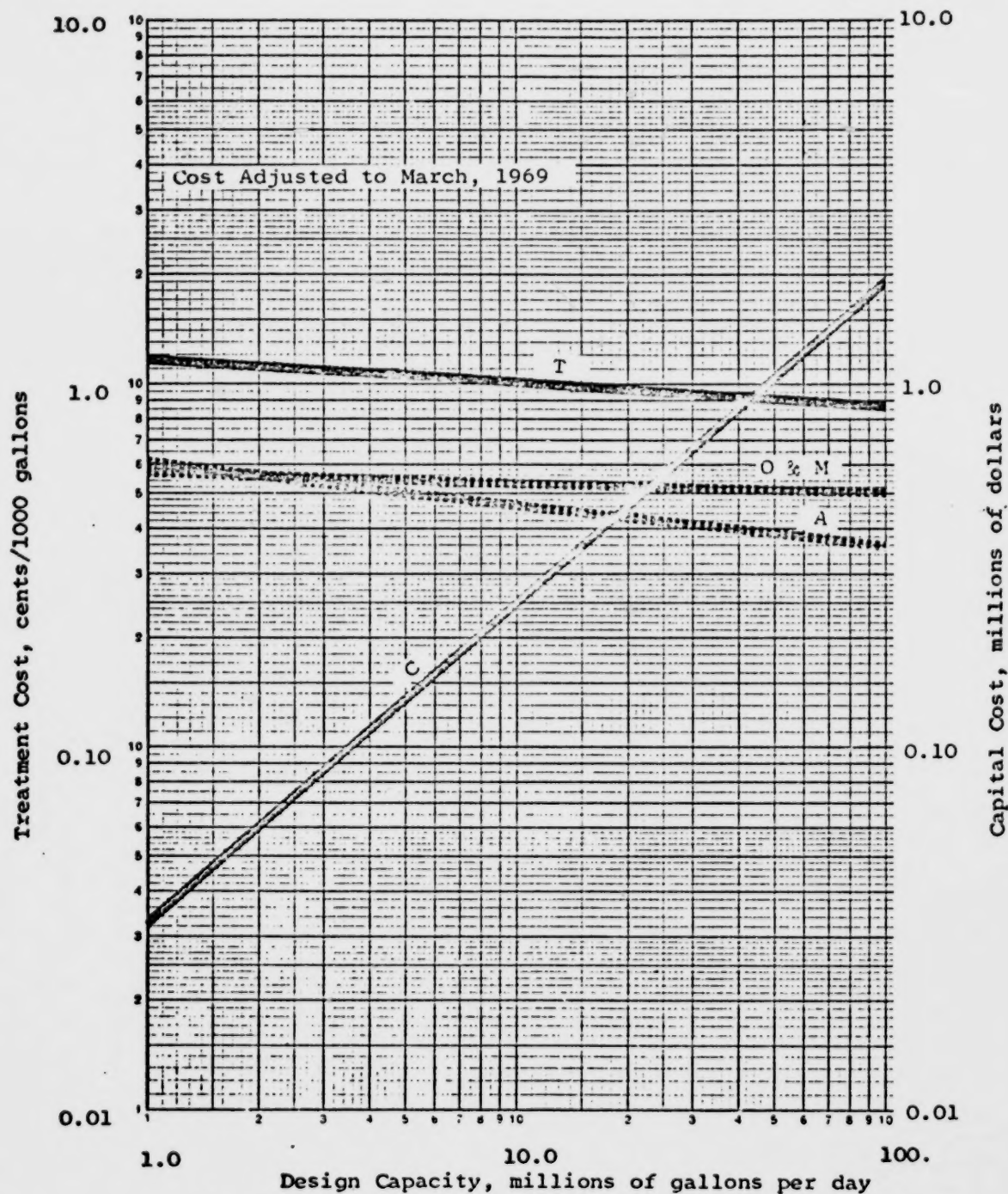


76" DIA. x 5' WIDE MACHINE



10' DIA. x 10' WIDE MACHINE

FIG. 3 MICROSCREENING OF SECONDARY EFFLUENT
Capital Cost, Operating & Maintenance Cost, Debt Service
vs.
Design Capacity



C = Capital Cost, millions of dollars
A = Debt Service, cents per 1000 gallons (4½% - 25 yr.)
O & M = Operating and Maintenance Cost, cents per 1000 gallons
T = Total Treatment Cost, cents per 1000 gallons

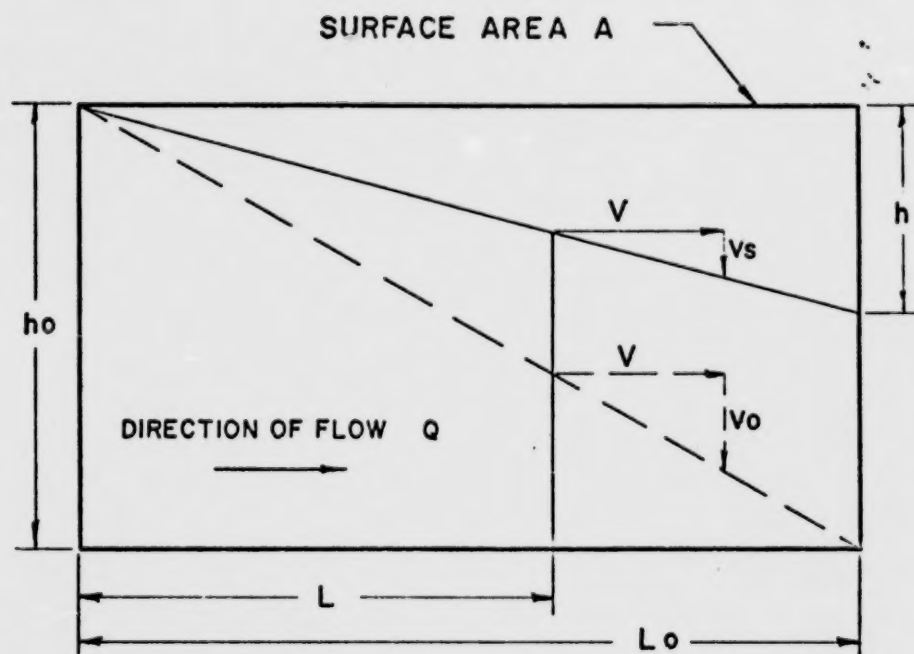


FIGURE 4
 IDEALIZED SETTLING PATHS OF DISCRETE PARTICLES IN A
 HORIZONTAL FLOW TANK

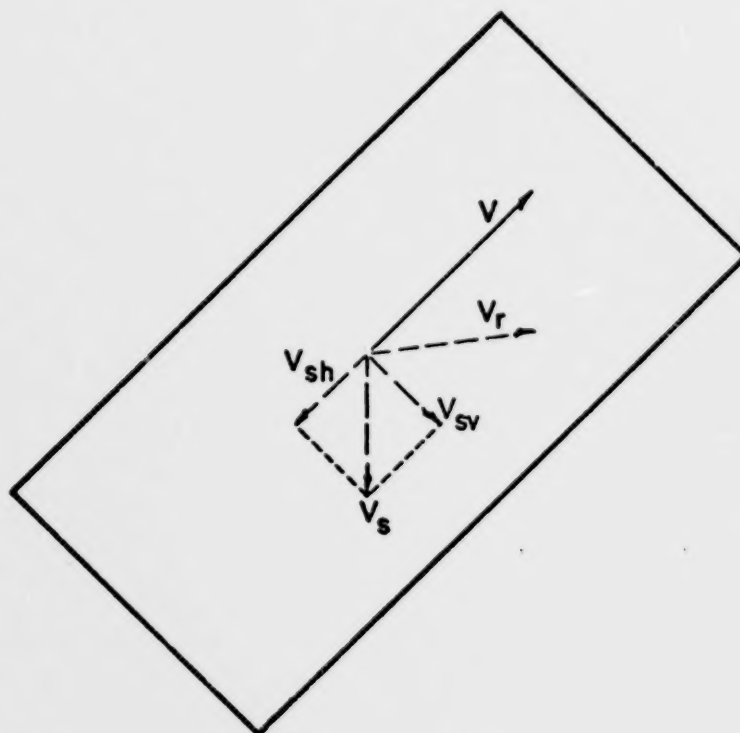


FIGURE 5
EFFECT OF TUBE INCLINATION ON SETTLING PATH OF
DISCRETE PARTICLE

FIGURE 6
MODULE OF STEEPLY
INCLINED TUBES

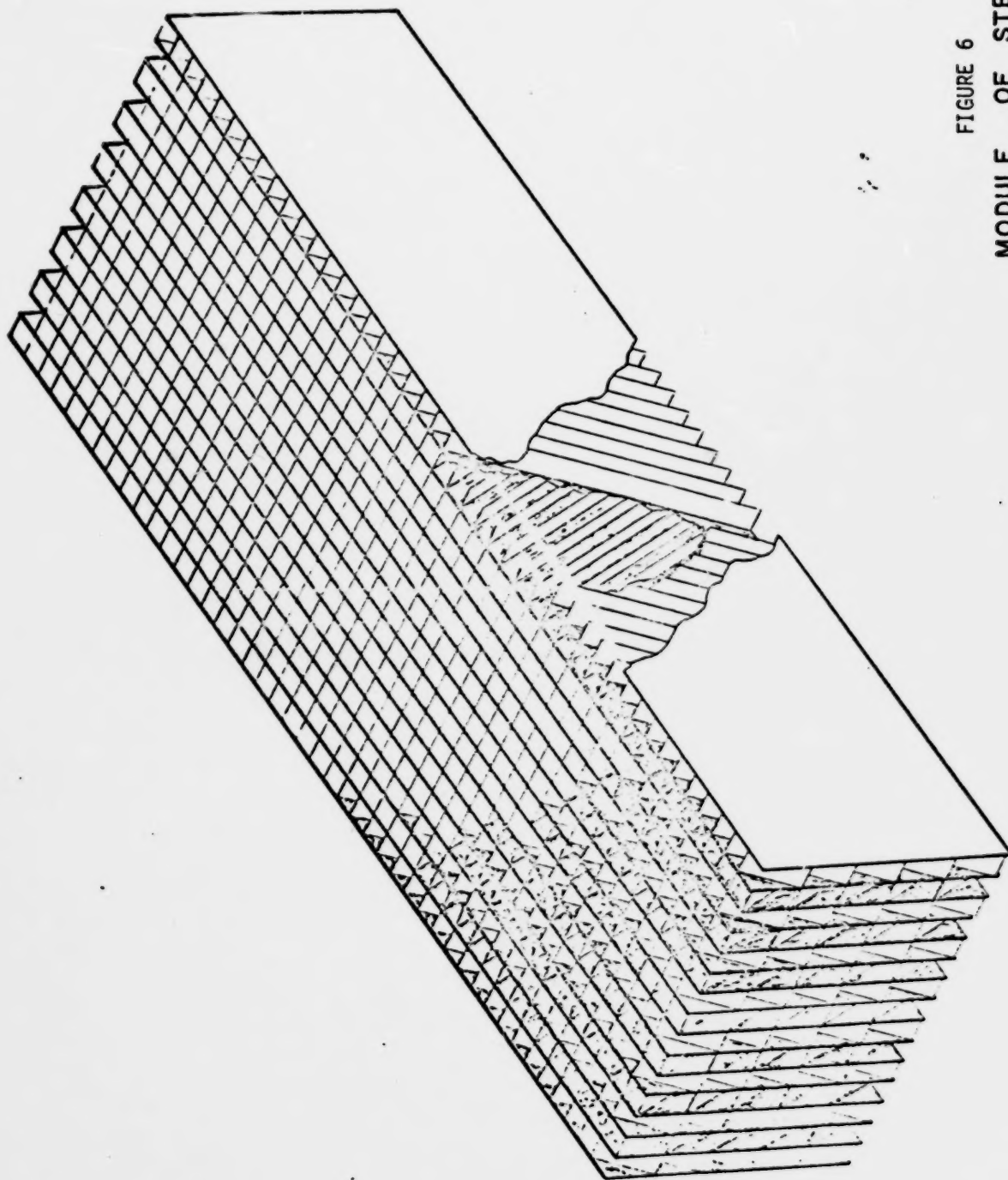
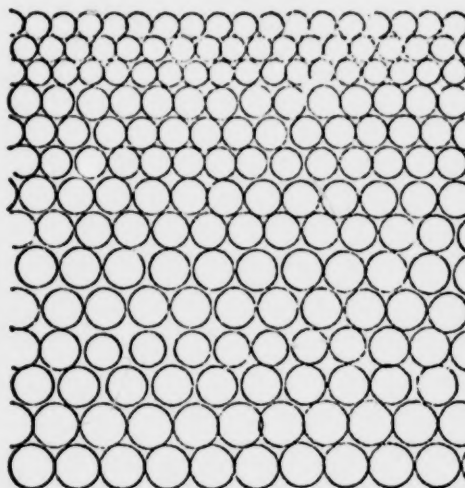
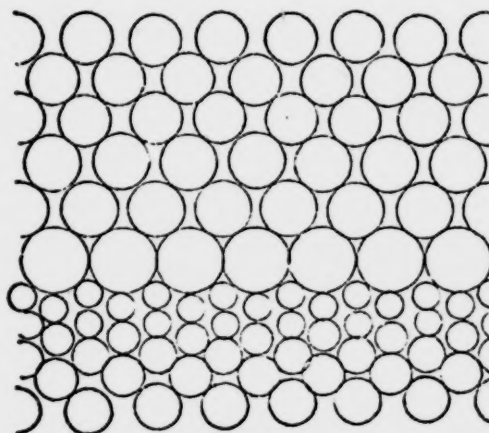


Fig. 7

Cross-Section Through
Single-Media Bed
Such as Conventional
Rapid Sand Filter



Cross-Section Through
Dual-Media Bed
Coarse Coal Above
Fine Sand



Cross-Section Through
Ideal Filter
Uniformly Graded From
Coarse to Fine
From Top to Bottom

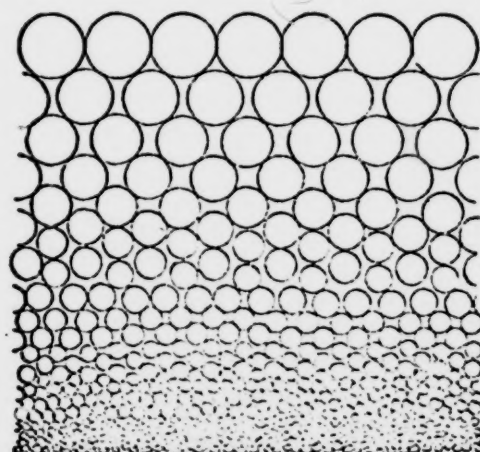
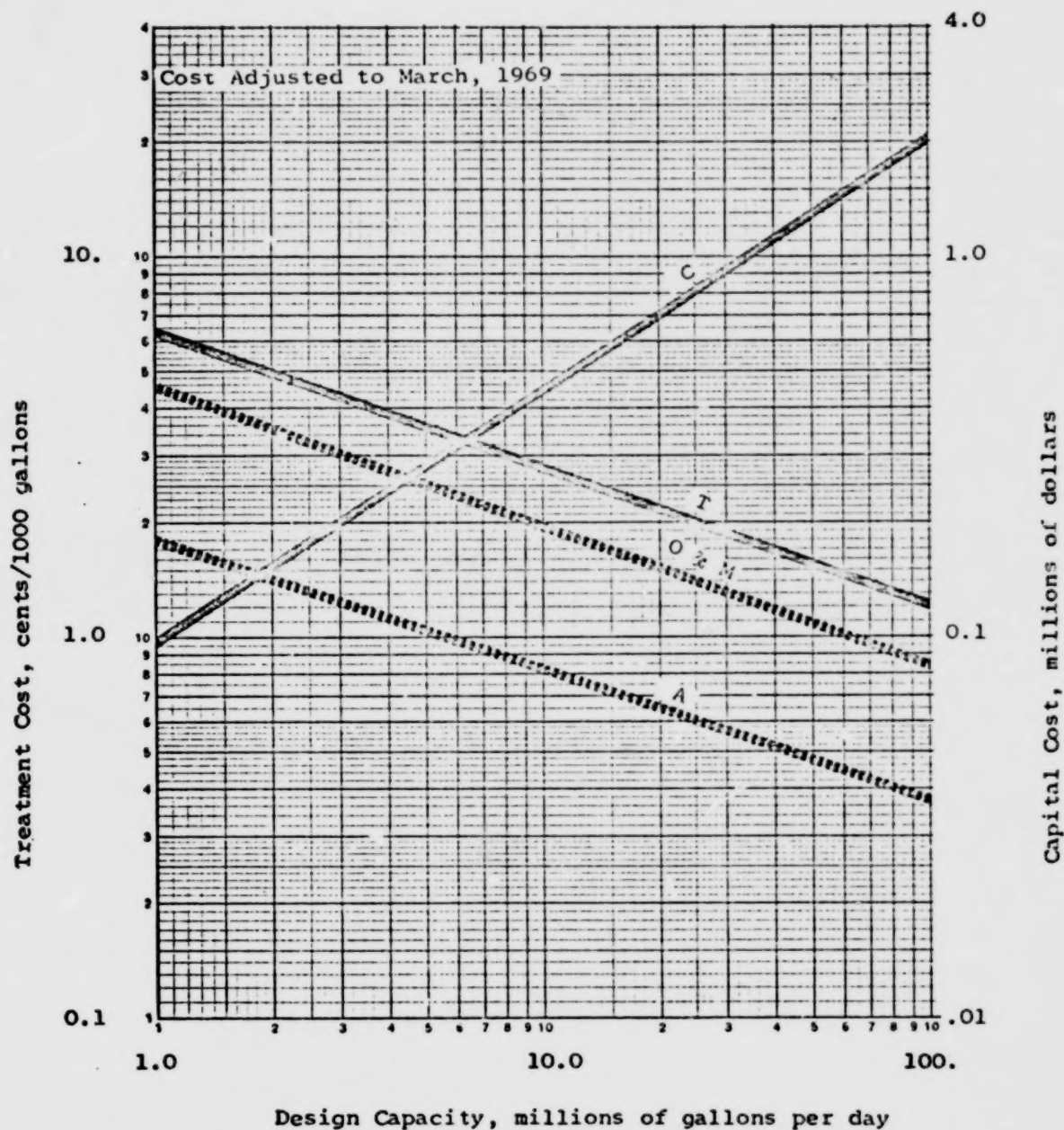


Fig. 8 FILTRATION THROUGH SAND OR GRADED MEDIA - 4GPM/SQ FT

Capital Cost, Operating & Maintenance Cost, Debt Service
vs.
Design Capacity

-20-



- C = Capital Cost, millions of dollars
- A = Debt Service, cents per 1000 gallons (4½% - 25 yr.)
- O & M = Operating and Maintenance Cost, cents per 1000 gallons
- T = Total Treatment Cost, cents per 1000 gallons